3.4.2 EXTERIOR SUPPLEMENTAL ELEMENTS

The construction of exterior supplemental moment frames, shear walls, or braced frames has many advantages. Exterior elements can be as effective in reducing loads on other elements as interior elements; yet, construction may be significantly less costly and access for equipment and materials will be significantly easier than for interior construction. Perhaps the single biggest advantage of exterior supplemental elements is that disruption of the functional use of the interior of the building will be minimized both during and after construction. Figure 3.4.2 shows the addition of an exterior supplemental concrete shear wall to an existing concrete or masonry building. Steel structures also can be used as buttresses.

There are, however, inherent problems in constructing supplemental exterior shear walls, braced frames, and moment frames. Many buildings do not have the necessary space to accommodate exterior structures due to the location of adjacent buildings or property lines. New exterior elements also may significantly affect the architectural aesthetics of the exterior of the building.

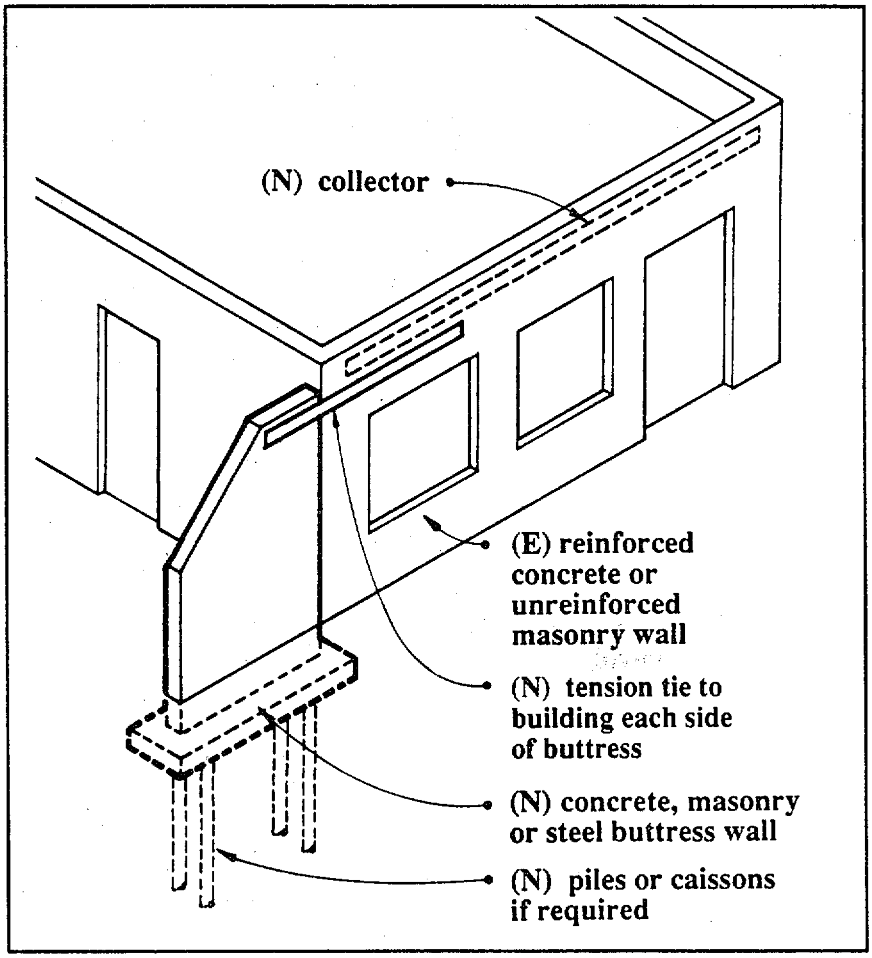

FIGURE 3.4.2 Example of supplemental in-plane strengthening by the addition of an external buttress.

Supplemental elements generally will require a significant capacity to resist overturning forces. Elements away from the building (e.g., the end of a buttress wall) will not be able to mobilize the dead weight of the building to resist the overturning forces, and significant uplift capacity therefore may be required in the new foundation.

The construction of exterior elements also does not preclude the need for interior construction. A load path must be provided to transfer forces from the existing building elements to the new external vertical-resisting elements. This usually necessitates the construction of collectors on the interior of the building.

3.4.3 INTERIOR SUPPLEMENTAL ELEMENTS

The construction of interior supplemental moment frames, shear walls, or braced frames will involve significant disruption of the functional operation of the building. Existing architectural coverings will need to be removed and new foundations constructed along with the new frame or wall and necessary collectors. It usually is desirable to locate new walls or frames along existing framing lines (i.e., framing into existing columns and beams) in order to provide boundary members, collectors, and dead load to help resist overturning forces while taking advantage of existing column foundations. Figure 3.4.3 shows the addition of a supplemental reinforced concrete shear wall on the interior of an existing concrete building. It should be noted that all concrete pours are subject to consolidation and shrinkage and, in this detail, the concrete may sag away from the underside of the concrete slab. This condition may be improved with proper mix design for low shrinkage or, alternatively, the lower wall can be made in two pours 48 hours apart. The initial pour would be up to about 18 inches from the slab soffit to allow sufficient space to form shear keys and to clean and prepare the surface for the following pour to the top of the slab.

Functional considerations likely will dictate the location of interior supplemental elements. This is particularly the case with shear walls or braced frames that will significantly break up the interior space.

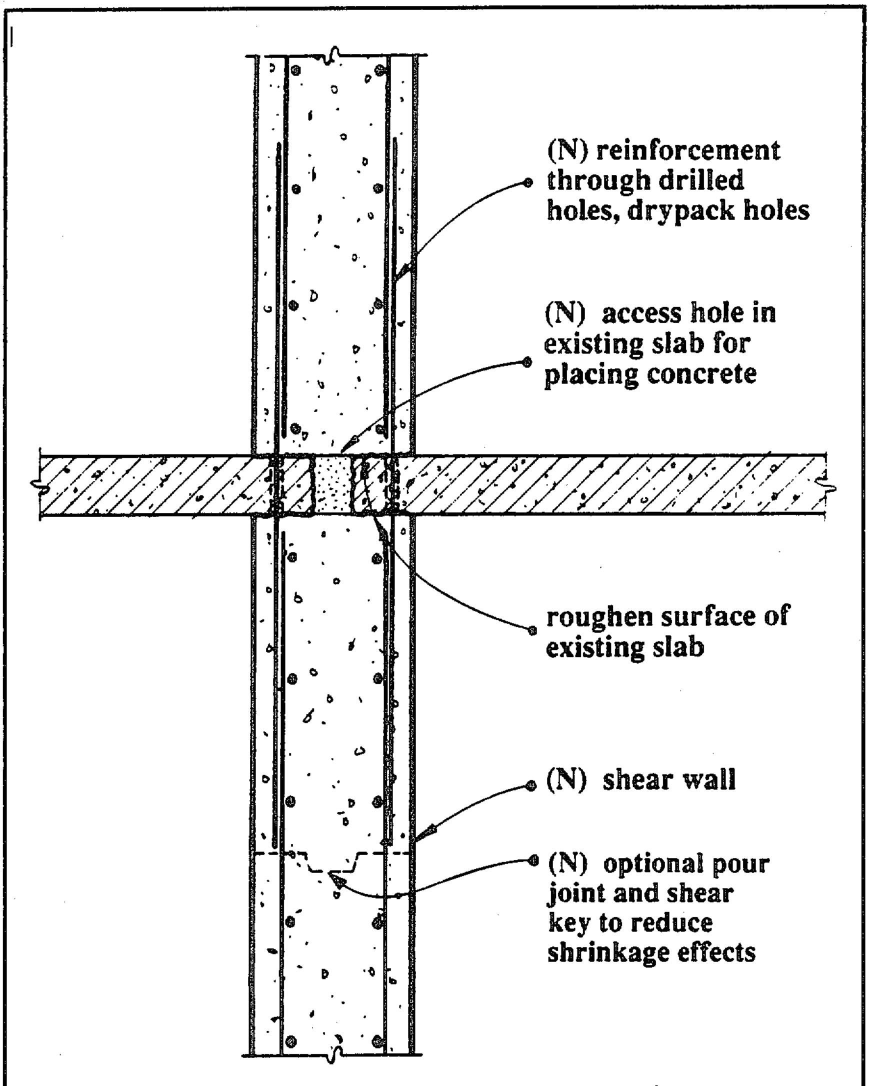

FIGURE 3.4.3 Connection of a supplemental interior shear wall.

3.5 DIAPHRAGMS

Diaphragms are horizontal subsystems that transmit lateral forces to the vertical-resisting elements. Diaphragms typically consist of the floors and roofs of a building. In this handbook, the term "diaphragm" also includes horizontal bracing systems. There are five principal types of diaphragms: timber diaphragms, concrete diaphragms, precast concrete diaphragms, steel decking diaphragms, and horizontal steel bracing.

Inadequate chord capacity is listed as a deficiency for most types of diaphragms. Theoretical studies, testing of diaphragms, and observation of earthquake-caused building damage and failures provide evidence that the commonly used method of determining diaphragm chord force (i.e., comparing the diaphragm to a flanged beam and dividing the diaphragm moment by its depth) may lead to exaggerated chord forces and, thus, overemphasize the need for providing an "adequate" boundary chord. Before embarking on the repair of existing chord members or the addition of new ones, the need for such action should be considered carefully with particular attention to whether the beam analogy is valid for calculating chord forces in the diaphragm under consideration.

Since few diaphragms have span-depth ratios such that bending theory is applicable, the capacity of the diaphragm to resist the tensile component of shear stress could be compared with tensile stresses derived from deep beam theory. In analyzing diaphragms by beam theory, chords provided by members outside of the diaphragms but connected to their edges may be considered and may satisfy the chord requirement.

3.5.1 TIMBER DIAPHRAGMS

3.5.1.1 Deficiencies

Timber diaphragms can be composed of straight laid or diagonal sheathing or plywood. The principal deficiencies in the seismic capacities of timber diaphragms are:

- Inadequate shear capacity of the diaphragm,
- Inadequate chord capacity of the diaphragm,
- Excessive shear stresses at diaphragm openings or at plan irregularities, and
- Inadequate stiffness of the diaphragm resulting in excessive diaphragm deformations.

3.5.1.2 Strengthening Techniques for Inadequate Shear Capacity

Techniques. Deficient shear capacity of existing timber diaphragms can be improved by:

1. Increasing the capacity of the existing timber diaphragm by providing additional nails or staples with due regard for wood splitting problems.
2. Increasing the capacity of the existing timber diaphragm by means of a new plywood overlay.
3. Reducing the diaphragm span through the addition of supplemental vertical-resisting elements (i.e., shear wall or braced frames) as discussed in Sec. 3.4.

Relative Merits. Adding nails and applying a plywood overlay (Techniques 1 and 2) require removal and replacement of the existing floor or roof finishes as well as removal of existing partitioning, but they generally are less expensive than adding new walls or vertical bracing (Technique 3). If the existing system consists of straight laid or diagonal sheathing, the most effective alternative is to add a new layer of plywood since additional nailing typically is not feasible because of limited spacing and edge distance. Additional nailing usually is the least expensive alternative, but the additional capacity is still limited to the number and capacity of the additional nails that can be driven (i.e., with minimum allowable end distance, edge distance, and spacing).

The additional capacity that can be developed by plywood overlays usually depends on the capacity of the underlying boards or plywood sheets to develop the capacity of the nails from the new overlay. Higher shear values are allowed for plywood overlay when adequate nailing and blocking (i.e., members with at least 2 inches of nominal thickness) can be provided at all edges where the plywood sheets abut. Adequate additional capacity for most timber diaphragms can be developed using this technique unless unusually large shears need to be resisted. When nailing into existing boards, care must be taken to avoid splitting. If boards are prone to splitting, pre-drilling may be necessary.

The addition of shear walls or vertical bracing in the interior of a building may be an economical alternative to strengthening the diaphragms particularly if the additional elements can be added without the need to strengthen the existing foundation. The alternative methodology described in Sec. 3.2.3.3 emphasizes control of the existing diaphragm response by cross walls or shear walls rather than by strengthening and, in that methodology, the shear transmitted to the in-plane walls is limited by the strength of the diaphragm. Although the methodology was developed for buildings with unreinforced masonry walls and flexible timber diaphragms, the above diaphragm provisions are considered to be generally applicable for timber diaphragms in buildings with other relatively rigid wall systems. When additional bracing or interior shear walls are required, relative economy depends on the degree to which ongoing operations can be isolated by dust and noise barriers and on the need for additional foundations.

3.5.1.3 Strengthening Techniques for Inadequate Chord Capacity

Techniques. Deficient diaphragm chord capacity can be improved by:

1. Providing adequately nailed or bolted continuity splices along joists or fascia parallel to the chord (Figure 3.5.1.3).

2. Providing a new continuous steel chord member along the top of the diaphragm.

3. Reducing the stresses on the existing chords by reducing the diaphragm's span through the addition of new shear walls or braced frames as discussed in Sec. 3.4.

Relative Merits. Wood diaphragms typically are constructed with minimal capacity to resist chord forces. Bottom wall plates nailed into the plywood are not spliced but butted; hence, the chord capacity provided at the bottom plate joints will be minimal. If the nailing between the bottom plate and the plywood is sufficient to transfer chord forces, splicing the top plate can be a means to provide this chord capacity. Steel straps can be nailed across the butted joint to provide this splice capacity, but notching of the bottom of some of the wood studs may be necessary to install the splice plates.

Another alternative is to utilize the double top plates on the wall below the diaphragm as the chord member. The double top plates typically are lapped and nailed. With sufficient lap nailing, the chord capacity of one plate can be developed if an adequate path for shear transfer is provided between the diaphragm and the top plates. This load path can be provided by nailing such as that shown in Figure 3.5.1.3. New or existing nailing needs to be verified or provided between the diaphragm sheathing, the edge blocking, the exterior sheathing, and the top plates.

Simplified calculations to determine stresses in diaphragm chords conservatively consider the diaphragm as a horizontal beam and ignore the flexural capacity of the web of the diaphragm as well as the effect of the out-of-plane shear walls that reduce the chord stresses. However, even though the chord requirements in some buildings may be overstated, in most buildings a continuous structural element is required at diaphragm boundaries to collect the diaphragm shears and transfer them to the individual resisting shear walls along each boundary (see Sec. 3.7.1).

A continuous steel member along the top of the diaphragm may be provided to function as a chord or collector member. For existing timber diaphragms at masonry or concrete walls, the new steel members may be used to provide wall anchorage as indicated in Figure 3.7.1.4b as wall as a chord or collector member for the diaphragm shear forces.

The lack of adequate chord capacity is seldom the reason why new shear walls or braced frames (Technique 3) would be considered to reduce the diaphragm loads. Reducing the diaphragm span and loads through the introduction of new vertical-resisting elements, however, may be considered to address other member deficiencies and, if so, the chord inadequacy problem also may be resolved.

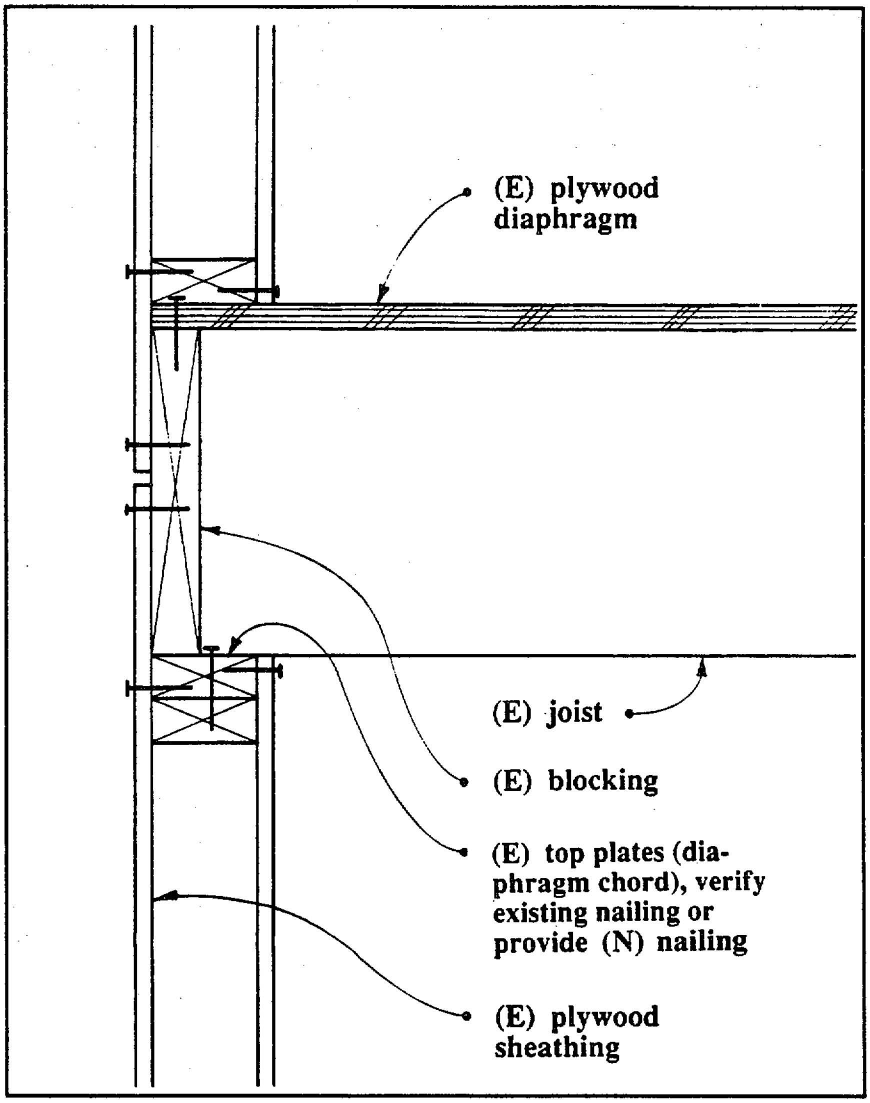

FIGURE 3.5.1.3 Exterior sheathing and top plate chord in a wood frame building.

3.5.1.4 Strengthening Techniques for Excessive Shear Stresses at Openings or Plan Irregularities

Techniques. Excessive shear stresses at diaphragm openings or other plan irregularities can be improved by:

1. Reducing the local stresses by distributing the forces along the diaphragm by means of drag struts (Figures 3.5.1.4a and 3.5.1.4b).

2. Increasing the capacity of the diaphragm by overlaying the existing diaphragm with plywood and nailing the plywood through the sheathing at the perimeter of the sheets adjacent to the opening or irregularity.

3. Reducing the diaphragm stresses by reducing the diaphragm spans through the addition of supplemental shear walls or braced frames as discussed in Sec. 3.4.

Relative Merits. The most cost-effective way to reduce large local stresses at diaphragm openings or plan irregularities is to install drag struts (Figures 3.5.1.4a and 3.5.1.4b), to distribute the forces into the diaphragm

(Technique 1). Proper nailing of the diaphragm into the drag struts is required to ensure adequate distribution of forces. Local removal of roof or floor covering will be required to provide access for nailing.

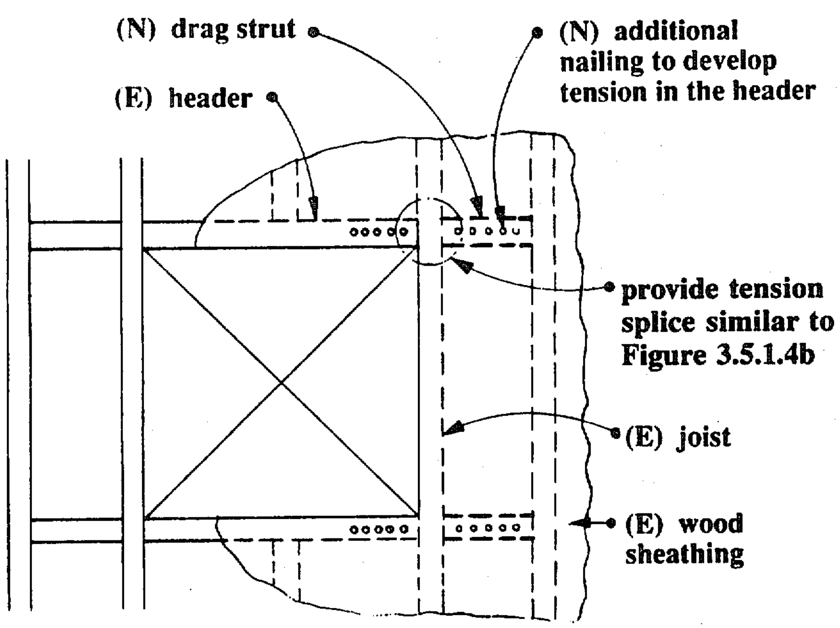

FIGURE 3.5.1.4a Reinforcement of an opening in an existing timber diaphragm.

The analysis for the design of the drag strut and the required additional nailing is similar to that for the reinforcement of an opening in the web of a steel plate girder. The opening divides the diaphragm into two parallel horizontal beams and the shear in each beam causes moment that induces tension or compression in the outer fibers of each beam. For small openings or low diaphragm shears, these bending forces may be adequately resisted as additional stresses in an existing diaphragm. For larger openings and/or larger diaphragms, tension or compression "flanges" may have to be developed at the opening. In a timber diaphragm, these "flanges" may be assumed to be the joists or headers that frame the opening, but to preclude distress due to stress concentration at the corners, the joists or headers must be continuous beyond the edge of the opening in order to transfer the flange forces back into the diaphragm by additional nailing.

Applying a plywood overlay (Technique 2) to increase the local diaphragm capacity or providing supplemental vertical-resisting elements (Technique 3) to reduce the local stresses generally will be viable alternatives only if they are being considered to correct other structural deficiencies.

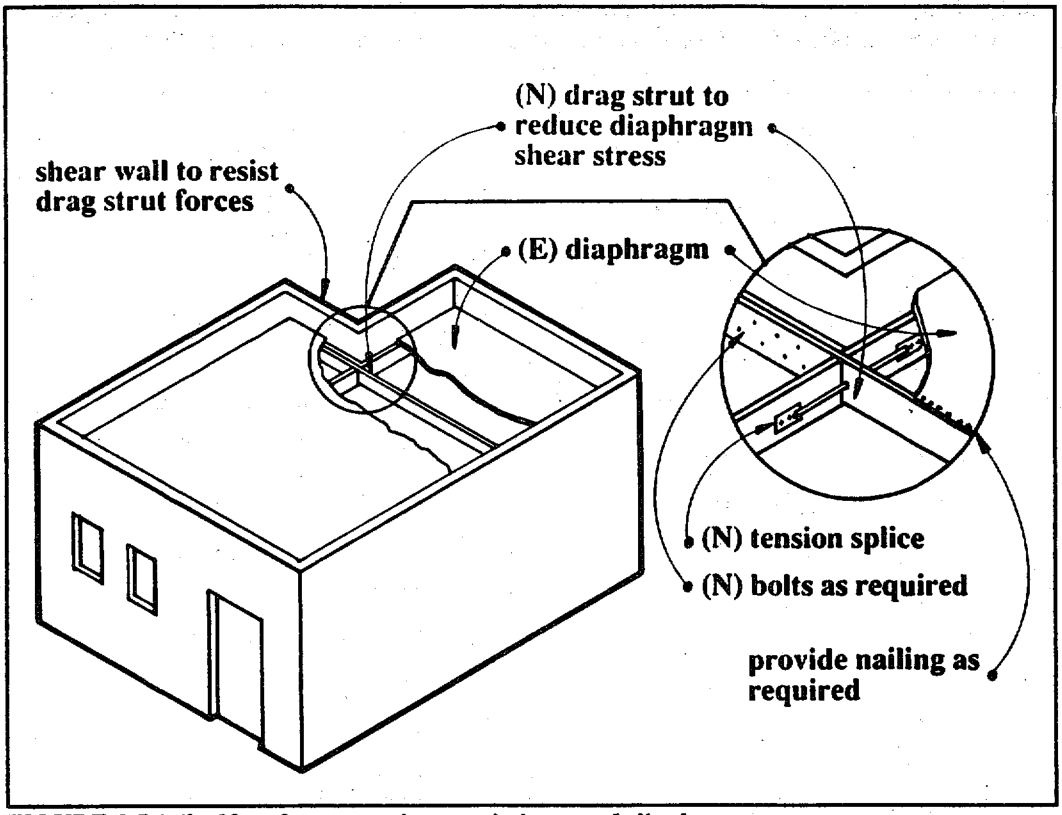

FIGURE 3.5.1.4b New drag strut in an existing wood diaphragm.

3.5.1.5 Strengthening Techniques for Inadequate Stiffness

Techniques. Excessive seismic displacement of an existing timber diaphragm can be prevented by:

1. Increasing the stiffness of the diaphragm by the addition of a new plywood overlay.

2. Reducing the diaphragm span and, hence, reducing the displacements by providing new supplemental vertical-resisting elements such as shear walls or braced frames as discussed in Sec. 3.4.

Relative Merits. The addition of new shear walls or braced frames (Technique 2) may be the most cost-effective alternative for reducing excessive displacements of plywood diaphragms (as is also the case for reducing excessive shear stresses as discussed above) if the additional elements can be added without strengthening the existing foundations and when the existing functional use of the building permits it.

The spacing of new vertical elements required to limit the deflection of straight or diagonal sheathing to prescribed limits may be too close to be feasible. In these cases, overlaying with plywood (Technique 1) may be the most cost-effective alternative. For timber diaphragms in buildings with rigid masonry or concrete walls, the alternative methodology described in Sec. 3.2.3.3 permits the use of sheathed timber cross walls to control the excessive displacements of an existing diaphragm as an alternative to strengthening.

3.5.2 CONCRETE DIAPHRAGMS

3.5.2.1 Deficiencies

The principal deficiencies of monolithic concrete diaphragms (i.e., reinforced concrete or post-tensioned concrete diaphragms) are:

- Inadequate in-plane shear capacity of the concrete diaphragm,
- Inadequate diaphragm chord capacity, and
- Excessive shear stresses at the diaphragm openings or plan irregularities.

3.5.2.2 Strengthening Techniques for Inadequate Shear Capacity

Techniques. Deficient in-plane shear capacity of monolithic concrete diaphragms can be improved by:

1. Increasing the shear capacity by overlaying the existing concrete diaphragm with a new reinforced concrete topping slab (Figure 3.5.2.2).
2. Reducing the shear in the existing concrete diaphragm by providing supplemental vertical-resisting elements (i.e., shear walls or braced frames) as discussed in Sec. 3.4.

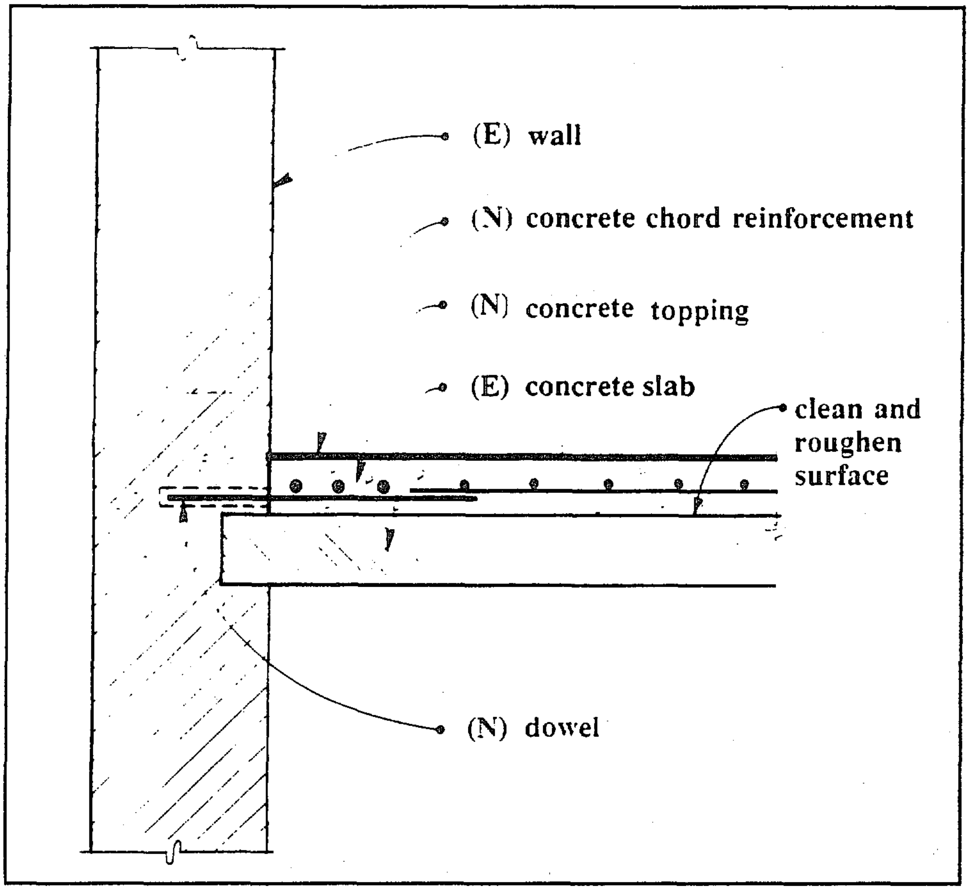

FIGURE 3.5.2.2 Strengthening an existing concrete diaphragm with a new topping slab and chord.

Relative Merits. Concrete diaphragms usually are strengthened with a concrete overlay. This will require removal and replacement of the existing partitions and floor finishes and will be disruptive to ongoing operations even though the work can be limited to one floor or a portion of a floor at a time. Adding the concrete overlay also will increase the dead weight of the structure; therefore, existing members, connections, and foundations must be checked to ensure that they are capable of resisting these added loads.

It may be possible to avoid strengthening a concrete diaphragm by providing additional shear walls or vertical bracing that will reduce the diaphragm shears. This alternative generally is more costly than the overlay, but it may be competitive when it can be restricted to the perimeter of the building and when minimal work is required on the foundations. For shear transfer, new reinforced concrete or masonry shear walls will require dowels grouted in holes drilled in the concrete diaphragms. When the concrete diaphragm is supported on steel framing, shear walls or vertical-bracing may be located under a supporting beam. Dowels or other connections for shear walls or bracing may be welded to the steel beam, but it also may be necessary to provide additional shear studs, welded to the steel beam, in holes drilled in the diaphragm slab to facilitate the shear transfer from the concrete slab to the steel beam.

3.5.2.3 Strengthening Techniques for Inadequate Flexural Capacity

Techniques. Deficient flexural capacity in monolithic concrete diaphragms can be improved by:

1. Increasing the flexural capacity by removing the edge of the diaphragm slab and casting a new chord member integral with the slab (Figure 3.5.2.3).

2. Adding a new chord member by providing a new reinforced concrete or steel member above or below the slab and connecting the new member to the existing slab with drilled and grouted dowels or bolts as discussed in Sec. 3.5.4.3.

3. Reducing the existing flexural stresses by providing supplemental vertical-resisting elements (i.e., shear walls or braced frames) as discussed in Sec. 3.4.

Relative Merits. If the existing concrete slab is supported on steel framing, the most cost-effective means of providing sufficient diaphragm chord capacity is to ensure adequate shear transfer of the diaphragm to the perimeter steel beam by adding drilled and grouted bolts and to ensure adequate strength and stiffness capacity of the perimeter beam connections. If a new chord is being secured with drilled and grouted anchors to an existing diaphragm containing prestressing strands, drilling must be done very carefully to ensure that the strands are not cut.

Figure 3.5.2.3 shows the provision of a new diaphragm chord and/or collector member as well as new dowels for wall anchorage or shear transfer from the existing concrete diaphragm. Because of the potential risk of gravity load failure at the interface with the existing slab, this detail is recommended only for one-way slabs in the direction parallel to the slab span. For other conditions, a detail using new concrete above or below the slab is recommended. Steel plates or shapes (as shown in Sec. 3.5.4.3) could be used with through bolts tightened to transfer load by friction.

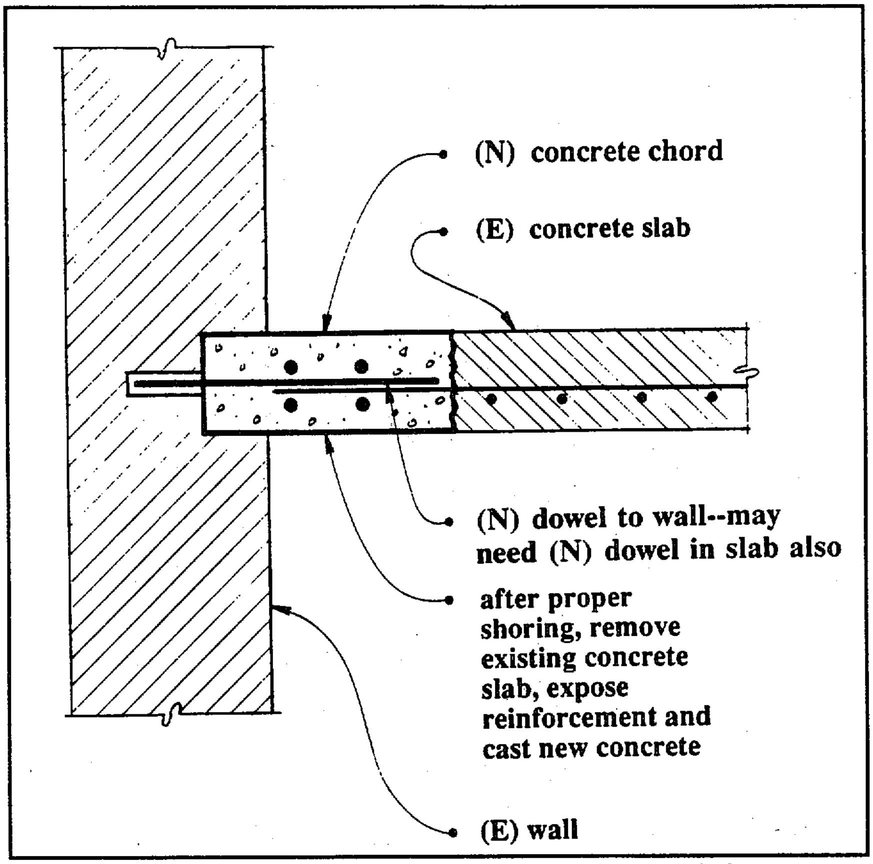

FIGURE 3.5.2.3 Adding a new chord member to an existing concrete diaphragm (not recommended for precast elements).

Providing new structural steel or reinforced concrete elements to reinforce the existing diaphragm at the openings is similar to the analysis described in Sec. 3.5.1.4. The tensile or compressive stresses in the new elements at the opening must be developed by shear forces in the connection to the existing slab. The new elements also must be extended beyond the opening a sufficient distance to transfer the tensile or compressive chord forces back into the existing slab in the same manner. Removing the stress concentration by filling in the opening (Technique 3) may be a feasible alternative provided that the functional requirements for the opening (e.g., stair or elevator shaft or utility trunk) no longer exist or it has been relocated.

3.5.2.4 Strengthening Techniques for Excessive Shear Stresses at Openings

Techniques. Deficient shear stress at diaphragm openings or plan irregularities in monolithic concrete slabs can be improved by:

1. Reducing the local stresses by distributing the forces along the diaphragm by means of structural steel (Figure 3.5.2.4a), or reinforced concrete elements cast beneath the slab and made integral through the use of drilled and grouted dowels.

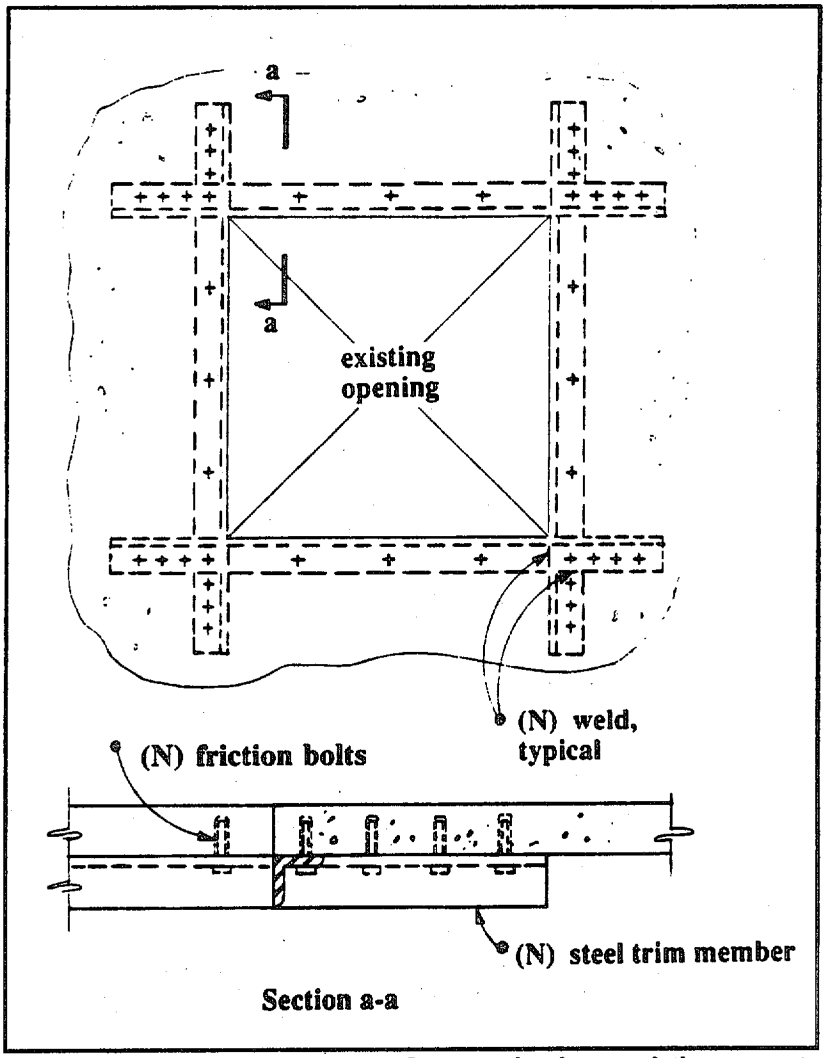

FIGURE 3.5.2.4a Reinforcement of an opening in an existing concrete diaphragm.

2. Increasing the capacity of the concrete by providing a new concrete topping slab in the vicinity of the opening and reinforcing with trim bars (Figure 3.5.2.4b).

3. Removing the stress concentration by filling in the diaphragm opening with reinforced concrete as indicated for shear walls in Figure 3.2.1.2b.

4. Reducing the shear stresses at the location of the openings by adding supplemental vertical-resisting elements (i.e., shear walls or braced frames) as discussed in Sec. 3.4.

Relative Merits. In existing reinforced concrete diaphragms with small openings or low diaphragm shear stress, the existing reinforcement may be adequate. If additional reinforcement is required, Technique 2 (i.e., new trim bars) probably will be the most cost-effective if a new topping slab is required to increase the overall diaphragm shear capacity.

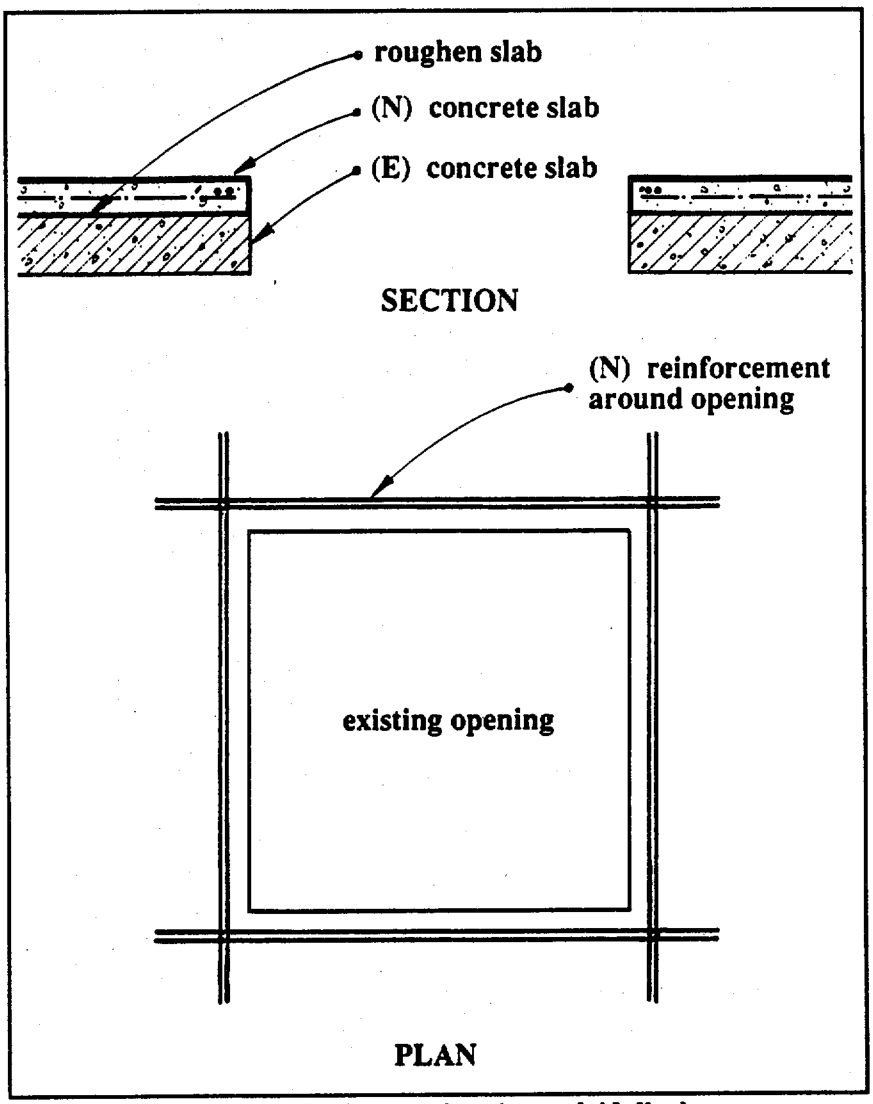

FIGURE 3.5.2.4b Strengthening openings in overlaid diaphragms.

3.5.3 POURED GYPSUM DIAPHRAGMS

3.5.3.1 Deficiencies

Poured gypsum diaphragms may be reinforced or unreinforced and may have the same deficiencies as monolithic concrete diaphragms (see Sec. 3.5.2.1.).

3.5.3.2 Strengthening Techniques for Poured Gypsum Diaphragms

Techniques. Strengthening techniques for deficiencies in poured gypsum diaphragms are similar to those listed for concrete diaphragms (see Sec. 3.5.2.2, 3.5.2.3, and 3.5.2.4); however, the addition of a new horizontal bracing system may be the most effective strengthening alternative.

Relative Merits. Poured gypsum has physical properties similar to those of very weak concrete. Tables of allowable structural properties (i.e., shear, bond, etc.) are published in various building codes and engineering manuals. A typical installation is for roof construction using steel joists. Steel bulb tees, welded or clipped to the joists, span over several joists and support rigid board insulation on the tee flanges. Reinforced or unreinforced gypsum is poured on the insulation board to a depth of 2 or 3 inches, embedding the bulbed stems of the tees. While use of the strengthening techniques discussed for reinforced concrete diaphragms (i.e., reinforced overlays, additional chord reinforcement, etc.) is technically possible, application of these techniques generally is not practical because of the additional weight or low allowable stresses of gypsum. Since dead loads normally constitute a significant portion of the design loads for roof framing members, the addition of several inches of gypsum for a reinforced overlay probably will overstress the existing light steel framing. Similarly, the low allowable stresses for dowels and bolts will allow strengthening of only marginally deficient diaphragms. For these reasons, gypsum diaphragms found to have significant deficiencies may have to be removed and replaced with steel decking or may be strengthened with a new horizontal bracing system (see Figure 3.5.5.2b).

3.5.4 PRECAST CONCRETE DIAPHRAGMS

3.5.4.1 Deficiencies

The principal deficiencies of precast or post-tensioned concrete planks, tees, or cored slabs are:

- Inadequate in-plane shear capacity of the connections between the adjacent units,
- Inadequate diaphragm chord capacity, and
- Excessive in-plane shear stresses at diaphragm openings or plan irregularities.

3.5.4.2 Strengthening Techniques for Inadequate Connection Shear Capacity

Techniques. Deficient in-plane shear capacity of connections between adjacent precast concrete planks, tees, or cored slabs can be improved by:

1. Replacing and increasing the capacity of the existing connections by overlaying the existing diaphragm with a new reinforced concrete topping slab (Figure 3.5.4.2).

2. Reducing the shear forces on the diaphragm by providing supplemental vertical-resisting elements (i.e., shear walls or braced frames) as discussed in Sec. 3.4.

Relative Merits. The capacity of an existing diaphragm composed of precast concrete elements (i.e., cored slabs, tees, planks, etc.) generally is limited by the capacity of the field connections between the precast elements. It may be possible to modify these connections for a moderate increase in diaphragm capacity; however, it usually is not feasible to develop the full shear capacity of the precast units except with an adequately doweled and complete poured-in-place connection. This usually is very costly. Overlaying the existing precast system with a new reinforced concrete topping (Technique 1) is an effective procedure for increasing the shear capacity of the existing diaphragm. Because of the relatively low rigidity of the existing connections, the new topping should be designed to resist the entire design shear. Existing floor diaphragms with precast concrete elements may have a 2- or 3-inch poured-in-place topping with mesh reinforcement to compensate for the irregularities in precast elements. Applying an additional topping slab over the existing slab may be prohibitive because of the additional gravity and seismic loads that must be resisted by the structure. Where mechanical connections between units exist along with a topping slab, the topping slab generally will resist the entire load (until it fails) because of the relative rigidities; therefore, the addition of mechanical fasteners generally is ineffective. For the above reasons the most cost-effective alternative may be reducing the diaphragm shear forces through the addition of supplemental shear walls or braced frames.

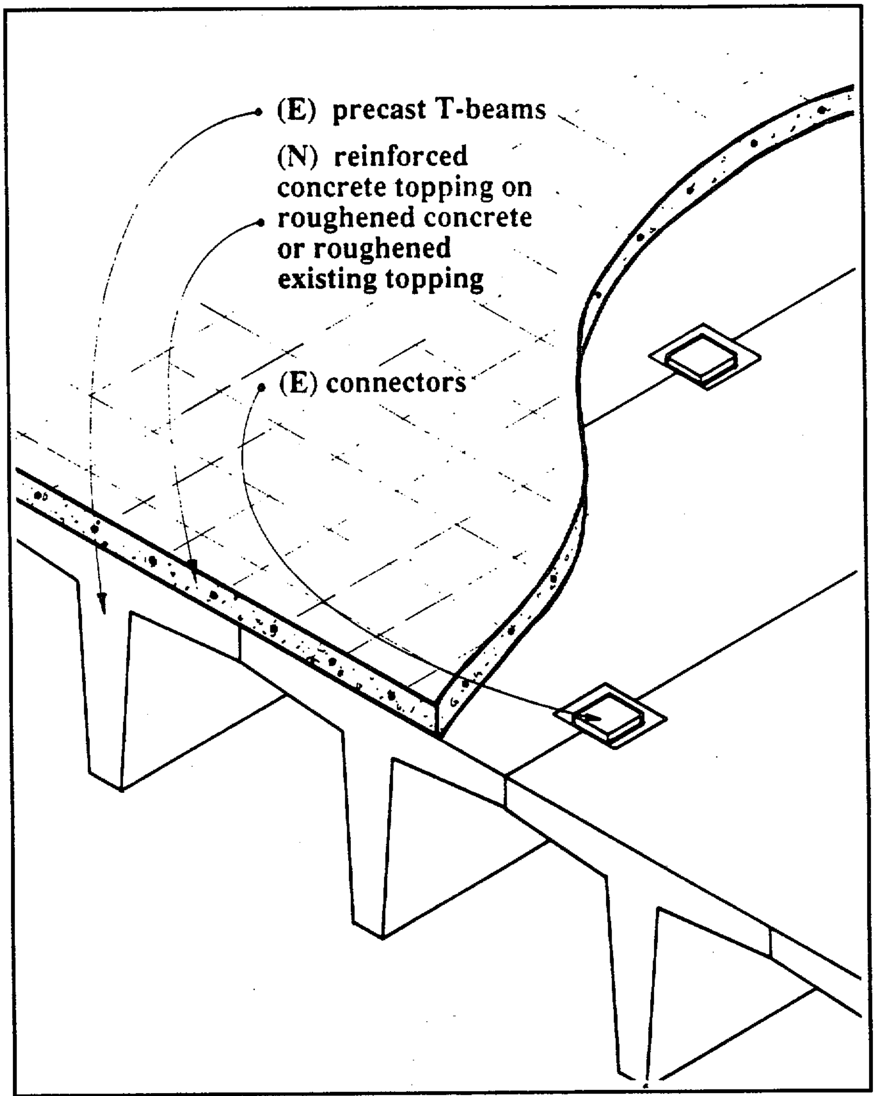

FIGURE 3.5.4.2 Strengthening an existing precast concrete diaphragm with a concrete overlay.

3.5.4.3 Strengthening Techniques for Inadequate Chord Capacity

Techniques. Deficient diaphragm chord capacity of precast concrete planks, tees, or cored slabs can be improved by:

1. Providing a new continuous steel member above or below the steel slab and connecting the new member to the existing slab with bolts (Figure 3.5.4.3).

2. Removing the edge of the diaphragm and casting a new chord member integral with the slab (Figure 3.5.2.3).

3. Reducing the diaphragm chord forces by providing supplemental vertical-resisting elements (i.e., shear walls or braced frames) as discussed in Sec. 3.4.

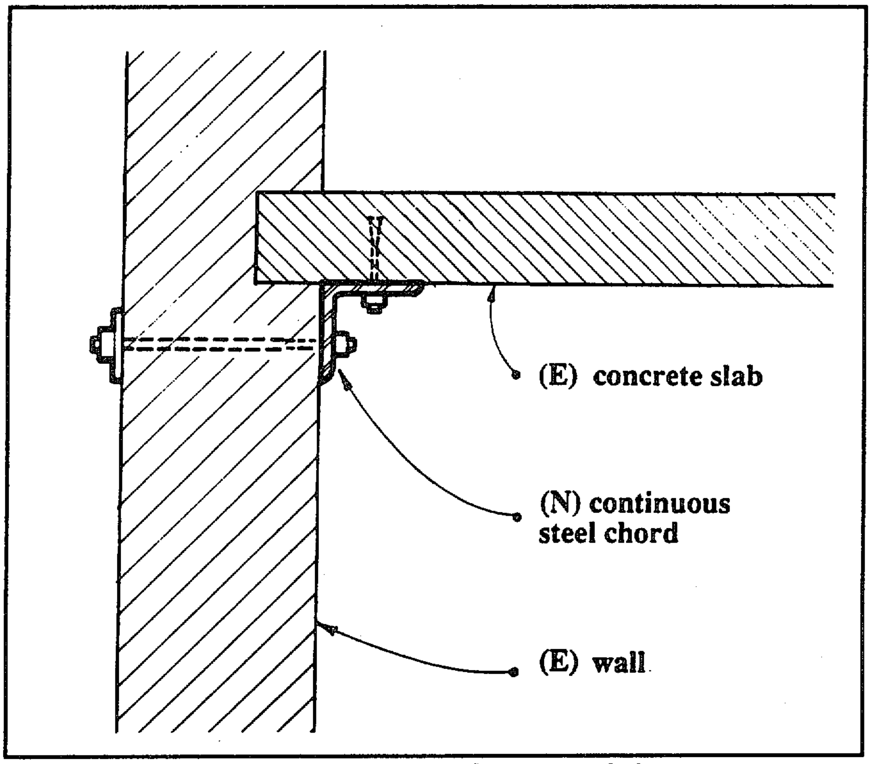

FIGURE 3.5.4.3 Adding a new steel member to an existing precast concrete diaphragm.

Relative Merits. Providing a new steel chord member (Technique 1) generally is the most cost-effective approach to rehabilitating a deficient diaphragm chord for precast concrete elements. When this approach is used; adequate shear transfer between the existing planks or slabs and the new chord member must be provided. Grouting under the new steel chord member may be necessary to accommodate uneven surfaces. Although typically more costly, casting a new chord into the diaphragm (Technique 2) may be considered a viable alternative where the projection caused by a new steel chord member is unacceptable for architectural reasons. If Technique 2 is considered, shoring of the planks or slabs will be necessary during construction. Technique 3 generally would be viable only if it is being considered to improve other deficient conditions.

3.5.4.4 Strengthening Techniques for Excessive Shear Stresses at Openings

Deficient diaphragm shear capacity at diaphragm openings or plan irregularities can be improved by:

1. Reducing the local stresses by distributing the forces along the diaphragm by means of concrete drag struts cast beneath the slab and made integral with the existing slab with drilled and grouted dowels.

2. Increasing the capacity by overlaying the existing slab with a new reinforced concrete topping slab with reinforcing trim bars in the vicinity of the opening (Figure 3.5.4.2).

3. Removing the stress concentration by filling in the diaphragm opening with reinforced concrete (Figure 3.2.1.2b).

4. Reducing the shear stresses at the location of the openings by providing supplemental vertical-resisting elements (i.e., shear walls or braced frames) as discussed in Sec. 3.4.

Relative Merits. The relative merits for rehabilitating excessive shear stresses at openings in precast concrete planks, tees, or core slabs are similar to those discussed in Sec. 3.5.2.4 for cast-in-place concrete diaphragms.

3.5.5 STEEL DECK DIAPHRAGMS

3.5.5.1 Deficiencies

The principal deficiencies in steel deck diaphragms are inadequate in-plane shear capacity which may be governed by the capacity of the welding to the supports or the capacity of the seam welds between the deck units, inadequate diaphragm chord capacity, and excessive in-plane shear stresses at diaphragm openings or plan irregularities.

3.5.5.2 Strengthening Techniques for Inadequate Shear Capacity

Deficient in-plane shear capacity of steel deck diaphragms can be improved by:

1. Increasing the steel deck shear capacity by providing additional welding.

2. Increasing the deck shear capacity of unfilled steel decks by adding a reinforced concrete fill (Figure 3.5.5.2a) or overlaying with concrete filled steel decks a new topping slab.

3. Increasing the diaphragm shear capacity by providing a new horizontal steel bracing system under the existing diaphragm (Figures 3.5.5.2b and 3.5.5.2d).

4. Reducing the diaphragm shear stresses by providing supplemental vertical-resisting elements to reduce the diaphragm span as discussed in Sec. 3.4.

Relative Merits. Steel decking, with or without an insulation fill (e.g., vermiculite or perlite), may be used as a diaphragm whose capacity is limited by the welding to the supporting steel framing and crimping or seam welding of the longitudinal joints of the deck units. The shear capacity of this type of diaphragm may be increased modestly by additional welding (Technique 1) if the shear capacity of the existing welds is less than the allowable shear of the steel deck itself. Significant increases in capacity may be obtained by adding a reinforced concrete fill (Technique 2) and shear studs welded to the steel framing through the decking. This procedure will require the removal of any insulation fill and the removal and replacement of any partitions and floor or roof finishes.

The shear capacity of steel deck diaphragms in open web joists often is limited by the lack of adequate connection from deck to shear wall or other vertical element. The lack of intermediate connectors between joists is common. Frequently, the joist bearing ends themselves are not well connected to transfer diaphragm shear. Addition of an edge support connected to wall and diaphragm often is feasible.

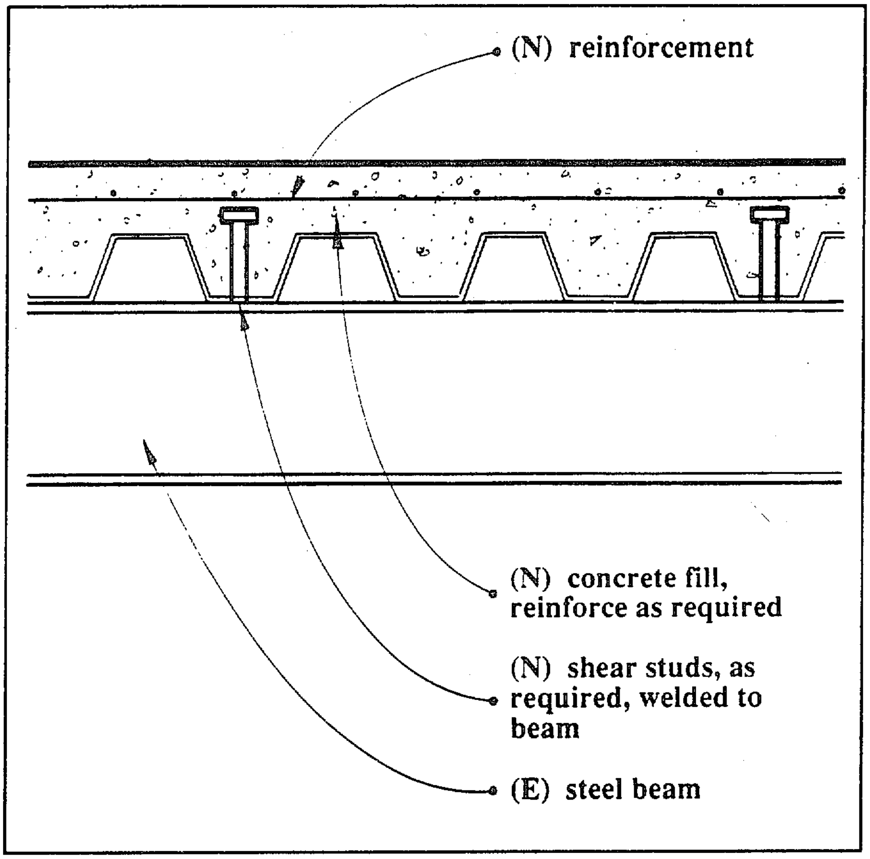

FIGURE 3.5.5.2a Strengthening an existing steel deck diaphragm.

The capacity of steel decking with an existing reinforced concrete fill may be increased by adding a reinforced concrete overlay (Technique 2). Although this is an expedient alternative for increasing the shear capacity of an existing composite steel deck, providing adequate shear transfer to the vertical-resisting members or chord elements through the existing composite decking may require special details (e.g., additional shear studs). Since the addition of a concrete overlay will increase the dead weight of the structure, the existing members, connections, and foundation must be checked to determine whether they are capable of resisting the added loads.

The above alternatives provide positive, direct methods for strengthening an existing steel deck diaphragm. Both alternatives require complete access to the top of the diaphragm and the removal and replacement of partitions and floor finishes. Technique 2 (i.e., topping over an existing concrete fill) will change the finished floor elevation by several inches and will therefore require a number of nonstructural adjustments to the new elevation (e.g., to stairs, elevators, floor electrical outlets, etc.).

An additional alternative for strengthening steel decking without concrete fill is to add new horizontal bracing under the decking (Technique 3). Since steel decking generally is supported on structural steel framing, the existing framing with new diagonal members forms the horizontal bracing system. The diaphragm shears are shared with the existing decking in proportion to the relative rigidity of the two systems. This alternative requires access to the underside of the floor or roof framing and may require relocation of piping, ducts, or electrical conduit as well as difficult and awkward connections to the existing framing. These costs must be weighed against the costs for a concrete overlay. It should be noted that this alternative may not be feasible for steel decking with a composite concrete fill because of the much greater rigidity of the existing composite diaphragm compared with that of the bracing system. For the bracing system to be effective in this case, the diaphragm shears would be distributed on the basis of the bracing system and the steel decking without the concrete fill (i.e., failure of the concrete fill in shear would be assumed to be acceptable). The new horizontal bracing system will require continuous chord or collector members (Figure 3.5.5.2d) to receive the bracing forces and transfer them to shear walls or other vertical-resisting elements. In Figure 3.5.5.2d, a tubular steel member is a preferred section for the new bracing members as is the tee section in Section a-a for the chord or collector members. Where existing construction does not permit the use of the tee section, an angle may be used as shown in Section b-b. In the latter case, bending of the angle and prying action on the anchor bolts may need to be investigated.

Reduction of the existing diaphragm stresses to acceptable levels by providing additional shear walls or vertical bracing (Technique 4) also may be a feasible alternative. The choice between shear walls or bracing will depend on compatibility with the existing vertical-resisting elements (i.e., additional shear walls should be considered for an existing shear wall system and additional bracing for an existing bracing system). The appropriateness of this technique (as discussed above) depends on the extent to which new foundations will be required and potential interference with the functional use of the building.

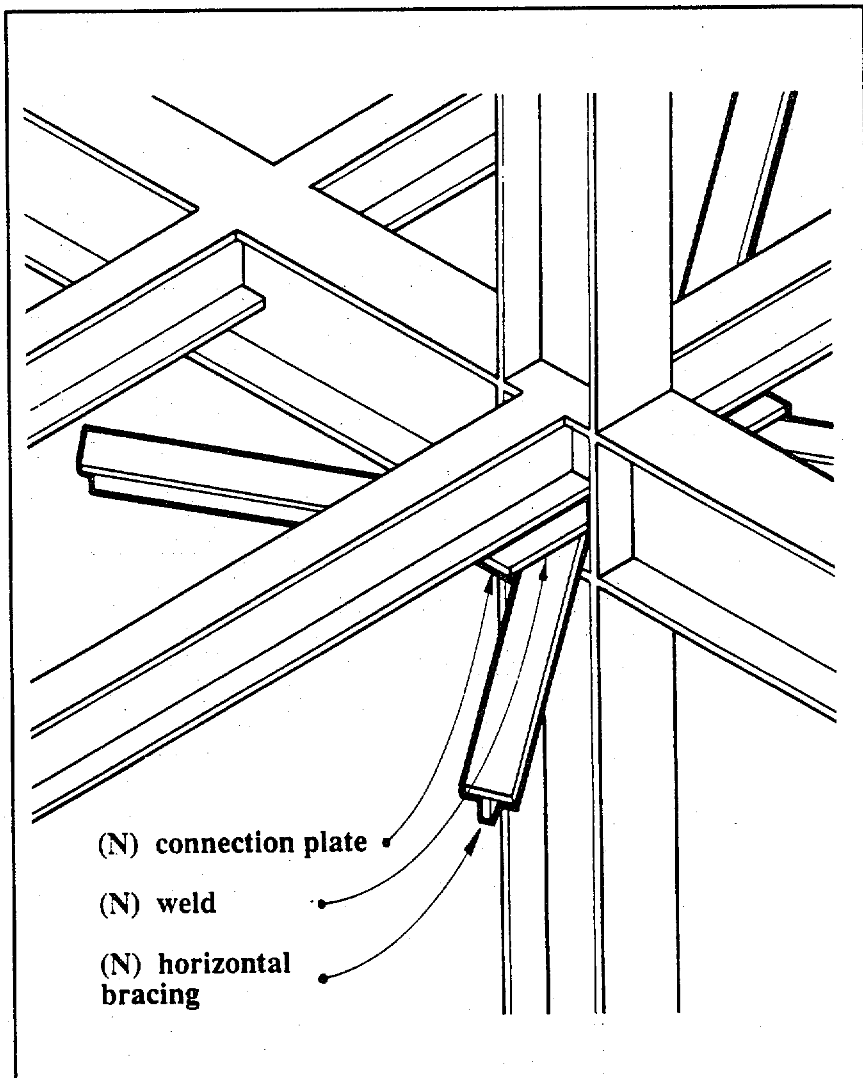

FIGURE 3.5.5.2b Strengthening an existing steel deck diaphragm.

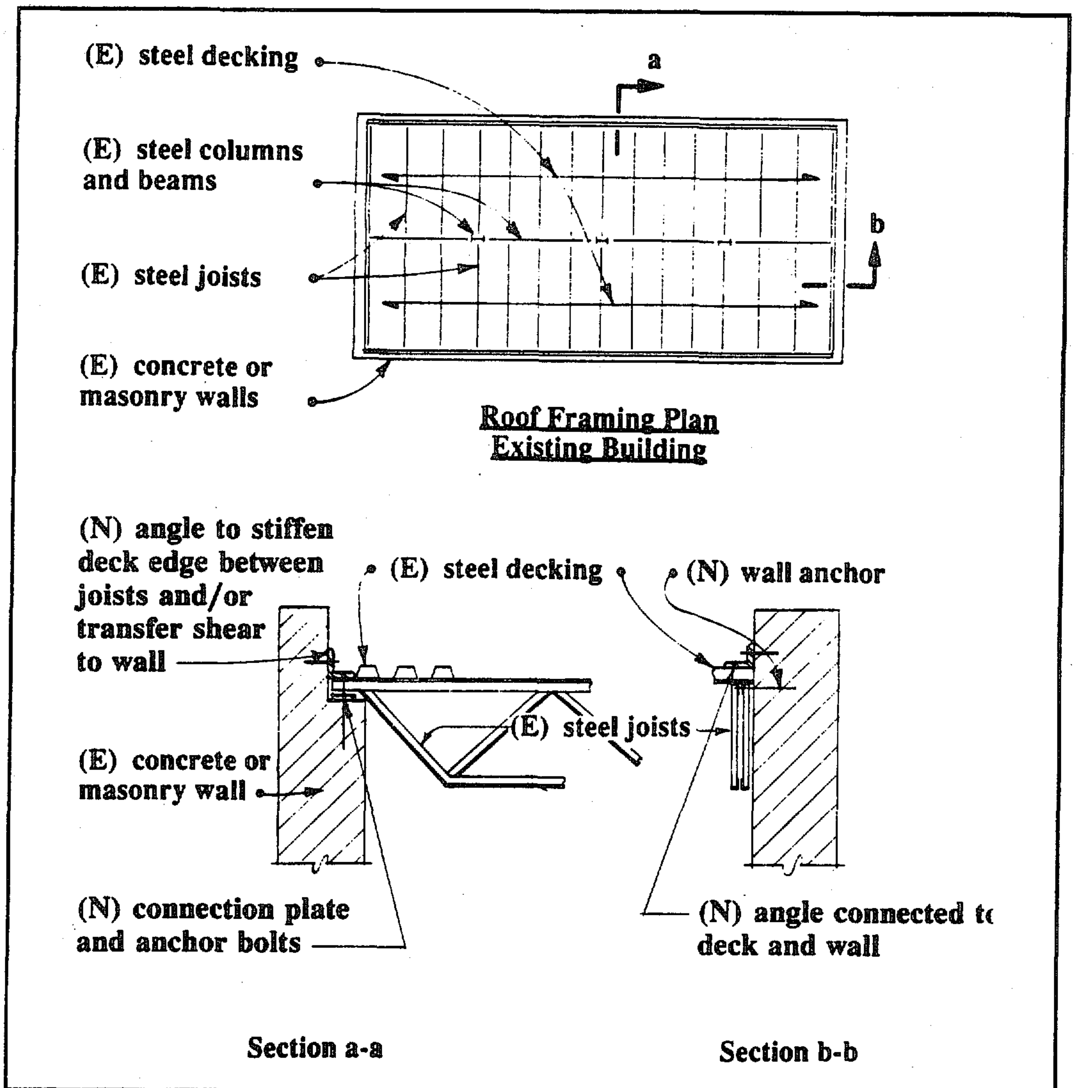

FIGURE 3.5.5.2c Strengthening an existing building with steel decking and concrete or masonry walls.

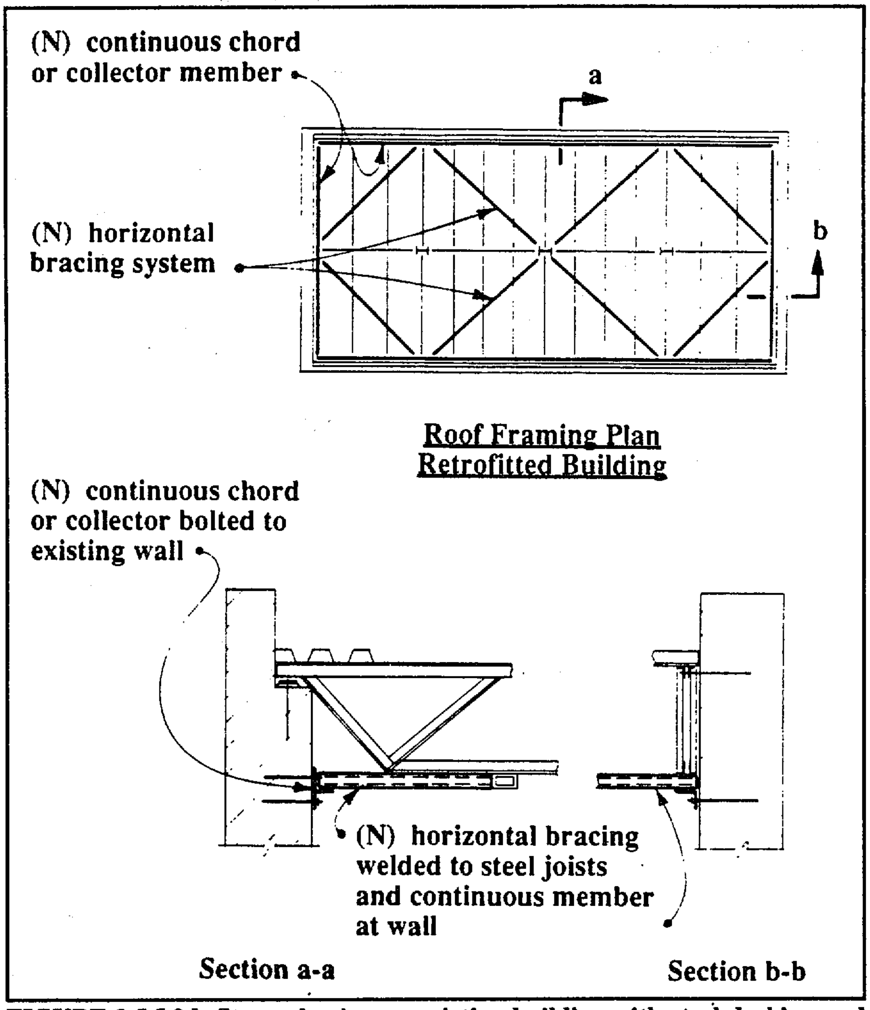

FIGURE 3.5.5.2d Strengthening an existing building with steel decking and concrete or masonry walls.

3.5.5.3 Strengthening Techniques for Inadequate Chord Capacity

Techniques. Deficient chord capacity of steel deck diaphragms can be improved by:

1. Increasing the chord capacity by providing welded or bolted continuity splices in the perimeter chord steel framing members.

2. Increasing the chord capacity by providing a new continuous steel member on top or bottom of the diaphragm (Figure 3.5.4.3).

3. Reducing the diaphragm chord stresses by providing supplemental vertical-resisting elements (i.e., shear walls or braced frames) such that the diaphragm span is reduced as discussed in Sec. 3.4.

Relative Merits. Steel decking generally is constructed on steel framing. The perimeter members of the steel framing typically will have sufficient capacity to resist the diaphragm chord stresses provided the shear capacity of the connections between the decking and the chord member and the tensile capacity of the steel framing connections are adequate to transfer the prescribed loads. Increasing the capacity of these connections by providing additional plug welds to the decking or adding steel shear studs in the case of concrete-filled metal decking may be required. Technique 1 generally is the most cost-effective.

Increasing the chord capacity by providing a new steel chord member to the perimeter of the diaphragm (Technique 2) would be appropriate only if it was impractical to use an existing member (Technique 1).

Reducing the diaphragm chord stresses by providing supplemental shear walls or braced frames (Technique 3) generally would not be cost-effective to correct a chord capacity problem unless it is being seriously considered to improve other component deficiencies as well.

3.5.5.4 Strengthening Techniques for Excessive Shear Stresses at Openings

Techniques. Excessive shear stresses at diaphragm openings or plan irregularities can be improved by:

1. Reducing the local stress concentrations by distributing the forces into the diaphragm by means of steel drag struts.

2. Increasing the capacity of the diaphragm by reinforcing the edge of the opening with a steel angle frame welded to the decking.

3. Reducing the diaphragm stresses by providing supplemental vertical-resisting elements (i.e., shear walls, braced frames or new moment frames) such that the diaphragm span is reduced as discussed in Sec. 3.4.

Relative Merits. Openings and plan irregularities in steel deck diaphragms generally are supported along the perimeter by steel beams. If continuous past the corners of the openings or irregularities, these beams can distribute the concentrated stresses into the diaphragm provided the capacity of the connections between the decking and the steel beams is adequate to transfer the prescribed loads. If inadequate, the connections can be reinforced by adding plug welds or shear studs.

If beams are not continuous beyond an opening or irregularity, new beams to act as drag struts can be provided (Technique 1). Adequate connection of the beams to the diaphragm and to the existing beams will be required to distribute loads.

Correcting the diaphragm deficiency by providing a steel frame around the perimeter of the opening or along the sides of the irregularity (Technique 2) is similar to providing drag struts. The connection between the new steel members and the diaphragm must be sufficient to adequately distribute the local stresses into the diaphragm. The dimensions of the opening or irregularity will dictate whether this can be achieved solely with the use of a perimeter steel frame.

Reducing the diaphragm stresses by providing supplemental shear walls or braced frames (Technique 3) generally would not be cost-effective to correct a diaphragm opening deficiency unless it also was being considered to improve other component deficiencies.

3.5.6 HORIZONTAL STEEL BRACING

3.5.6.1 Deficiency

The principal deficiency in horizontal steel bracing systems is inadequate force capacity of the members (i.e., bracing and floor or roof beams) and/or the connections.

3.5.6.2 Strengthening Techniques for Braces or Beams

Techniques. Deficient horizontal steel bracing system capacity can be improved by:

1. Increasing the capacity of the existing bracing members or removing and replacing them with new members and connections of greater capacity.

2. Increasing the capacity of the existing members by reducing unbraced lengths.

3. Increasing the capacity of the bracing system by adding new horizontal bracing members to previously unbraced panels (if feasible).

4. Increasing the capacity of the bracing system by adding a steel deck diaphragm to the floor system above the steel bracing.

5. Reducing the stresses in the horizontal bracing system by providing supplemental vertical-resisting elements (i.e., shear walls or braced frames) as discussed in Sec. 3.4.

Relative Merits. Horizontal bracing systems to resist wind or earthquake forces have been in common use for many years in steel-framed industrial buildings. These bracing systems generally are integrated with the existing floor or roof framing systems, and the capacity of the bracing system should be governed by the diagonal braces and their connections. If the structural analysis indicates that the existing floor or roof framing members in the bracing systems do not have adequate capacity for the seismic loads, providing additional bracing or other lateral-load-resisting elements may be a cost-effective alternative to strengthening these members.

Simple strengthening techniques include increasing the capacity of the existing braces and their connections (e.g., single-angle bracing could be doubled, double-angle bracing could be "starred") as well as removing existing braces and replacing them with stronger braces and connections (Technique 1). If the compressive capacity of the elements is the primary deficiency, providing a system of secondary braces that reduces the unbraced lengths (Technique 2) of the members may be cost-effective. The existing connections must be investigated and, if found to be inadequate, the connections will need to be strengthened. Technique 3 (providing horizontal braces in adjacent unbraced panels if present) may be a very cost-effective approach to increasing the horizontal load capacity.

Existing horizontal bracing systems often do not have an effective floor diaphragm and new floor or roof diaphragm consisting of a reinforced concrete slab or steel decking with or without concrete fill can be provided to augment or replace the horizontal bracing systems (Technique 4). A steel deck diaphragm may be designed to augment the horizontal bracing, but a concrete slab probably would make the bracing ineffective because of the large difference in rigidities. The concrete slab therefore would need to be designed to withstand the entire lateral load.

As with other diaphragms, it may be possible to reduce diaphragm stresses to acceptable limits by providing additional shear walls or vertical bracing (Technique 5). However, unlike true diaphragm systems, a horizontal bracing system may not have the same shear capacity at any section (e.g., a simple bracing system between two end walls may have increasing shear capacity from the center towards each end). In some cases, additional vertical-resisting elements can increase the stresses in some of the elements of the existing bracing systems.

3.6 FOUNDATIONS

Deficient foundations occasionally are a cause for concern with respect to the seismic capacity of existing buildings. Because the foundation loads associated with seismic forces are transitory and of very short duration, allowable soil stresses for these loads, combined with the normal gravity loads, may be permitted to approach ultimate stress levels. Where preliminary analysis indicates that there may be significant foundation problems, recommendations from a qualified geotechnical engineer should be required to establish rational criteria for the foundation analysis.

3.6.1 CONTINUOUS OR STRIP WALL FOOTINGS

3.6.1.1 Deficiencies

The principal deficiencies in the seismic capacity of existing continuous or strip wall footings are:

- Excessive soil bearing pressure due to overturning forces and
- Excessive uplift conditions due to overturning forces.

3.6.1.2 Strengthening Techniques for Excessive Soil Bearing Pressure

Techniques. The problem of excessive soil bearing pressure caused by seismic overturning forces can be mitigated by:

1. Increasing the bearing capacity of the footing by underpinning the footing ends and providing additional footing area (Figure 3.6.1.2a).

2. Increasing the vertical capacity of the footing by adding new drilled piers adjacent and connected to the existing footing (Figure 3.6.1.2b).

3. Increasing the soil bearing capacity by modifying the existing soil properties.

4. Reducing the overturning forces by providing supplemental vertical-resisting elements (i.e., shear walls or braced frames).

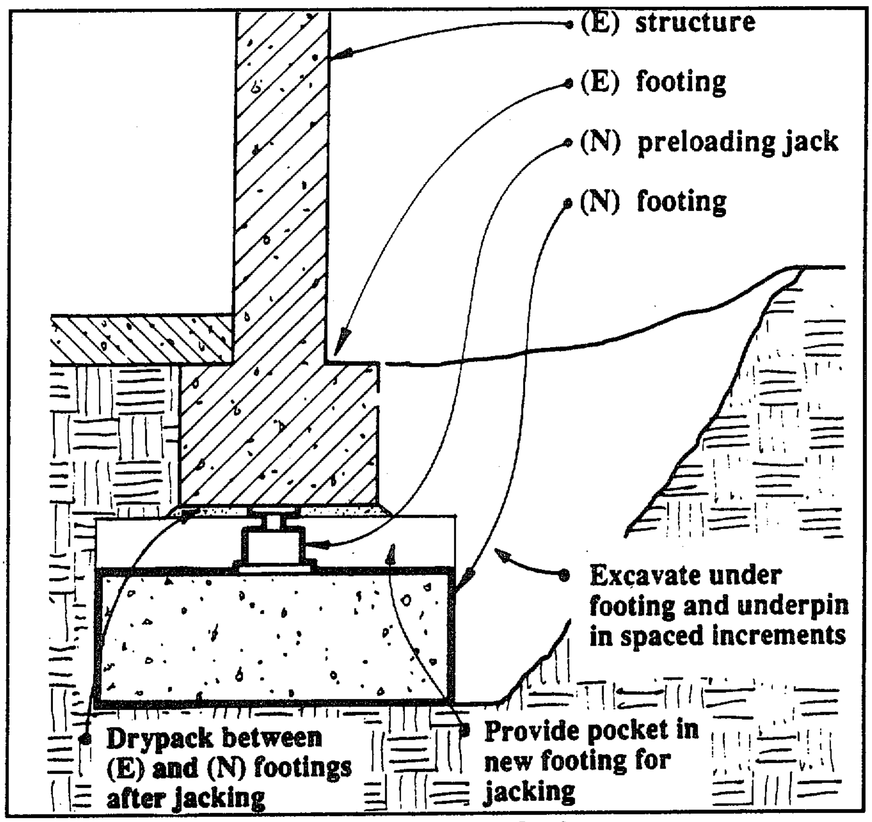

FIGURE 3.6.1.2a Underpinning an existing footing.

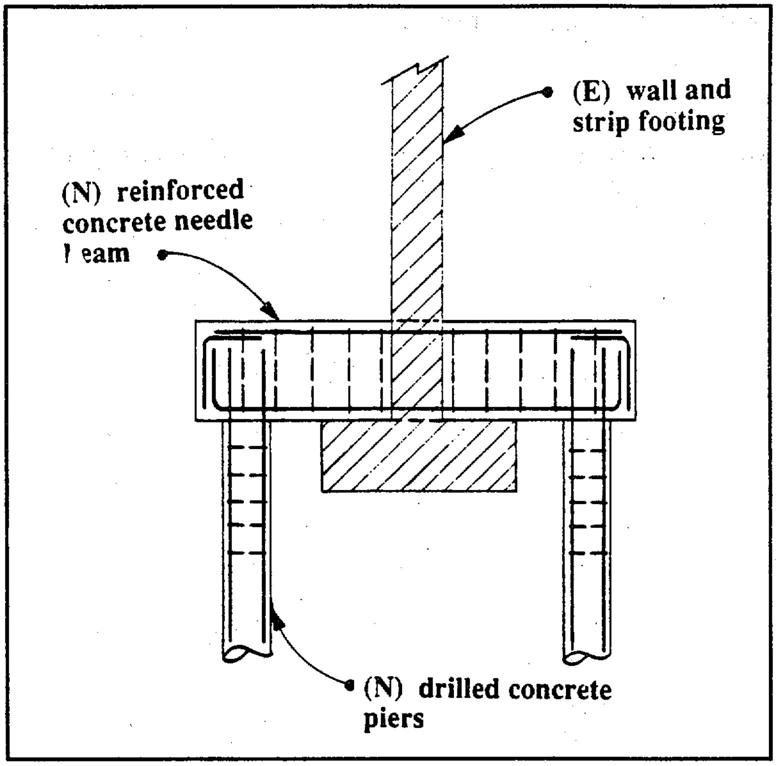

FIGURE 3.6.1.2b Strengthening an existing wall footing by the addition of drilled piers.

Relative Merits. The most effective procedure for correcting excessive soil pressure due to seismic overturning forces generally is to underpin the ends of the footing and to construct a larger footing under each end of the existing footing (Technique 1). The new footing should be constructed in staggered increments, and each increment should be preloaded by jacking prior to transfer of the load from the existing footing. An alternative procedure is to provide a drilled pier on each side and at each end of the wall (Technique 2). The reinforced concrete piers should be cast-in-place in uncased holes so as to develop both tension and compression. Each pier should extend above the bottom of the footing and be connected by a reinforced concrete beam through the existing wall above the footing (Figure 3.6.1.2.b).

Techniques 1 and 2 are costly and disruptive. For this reason, when seismic upgrading results in increased forces that require foundation strengthening, it may be cost-effective to consider other seismic upgrading schemes. Soil conditions may be such that modifying the capacity of existing soils is the most viable alternative. The soil beneath structures founded on clean sand can be strengthened through the injection of chemical grouts. The bearing capacity of other types of soils can be strengthened by compaction grouting. With chemical grouting, chemical grout is injected into clean sand in a regular pattern beneath the foundation. The grout mixes with the sand to form a composite material with a significantly higher bearing capacity. With compaction grouting, grout also is injected in a regular pattern beneath the foundation but it displaces the soil away from the pockets of injected grout rather than dispersing into the soil. The result of the soil displacement is a densification of the soil and, hence, increased bearing capacity. Some disruption of existing floors adjacent to the subject foundations may be required in order to cut holes needed for uniform grout injection. Alternatively, seismic forces on the footing can be reduced by adding other vertical-resisting elements such as bracing, shear walls, or buttresses.

3.6.1.3 Strengthening Techniques for Excessive Uplift Conditions

Techniques. Deficient capacity of existing foundations to resist prescribed uplift forces caused by seismic overturning moments can be improved by:

1. Increasing the uplift capacity of the existing footing by adding drilled piers or soil anchors.

2. Increasing the size of the existing footing by underpinning to mobilize additional foundation and soil weight.

3. Reducing the uplift forces by providing supplemental vertical-resisting elements (i.e., shear walls or braced frames) as discussed in Sec. 3.4.

Relative Merits. Any seismic rehabilitation alternative that requires significant foundation work will be costly. Access for heavy equipment (e.g., drilling rigs, backhoes, and pile drivers), ease of material handling, and the need to minimize the disruption of the functional use of the building are a few of the reasons why exterior foundation rehabilitation work will be significantly less costly than interior work.

Providing a significant increase in the uplift capacity of an existing foundation generally is most effectively achieved by adding drilled piers or soil anchors (Technique 1). Reinforced concrete piers can be provided adjacent to the footing and connected to the existing footing with steel or concrete beams (Figure 3.6.1.2b). Locating the piers symmetrically on both sides of the footing will minimize connections that must transfer eccentric loads. The details for eccentric connections may not always be feasible. However, providing concentric drilled piers almost ensures that interior foundation work will be needed.

Soil anchors similar to those used to tie-back retaining walls also can be used instead of drilled piers. Hollow core drill bits from 6 inches to 2 feet in diameter can be used to drill the needed deep holes. After drilling, a deformed steel tension rod is placed into the hole through the center of the bit. As the bit is withdrawn, cement grout is pumped through the stem of the bit bonding to the tension rod and the soil. These types of soil anchors can provide a significant tensile capacity. Drilling rigs are available that can drill in the interior of buildings even with low headroom; however, this is more costly.

Underpinning the ends of the footing to create a wider bearing area at each end has the beneficial effect of reducing the uplift by increasing the area, the moment of inertia, and the dead load of the existing footing. Although this may be a feasible alternative, it is usually less cost-effective than adding drilled piers or soil anchors. The size of the necessary footing addition becomes prohibitive if substantial uplift forces need to be resisted.

As with other rehabilitation techniques, reducing the overturning forces by providing additional vertical-resisting elements (Technique 3) such as braced frames, shear walls, or buttresses may be viable. The addition of buttresses may transfer loads to the exterior of the building where foundation work may not be so costly.

Some engineers believe that uplifting of the ends of rigid shear walls is not a deficiency and may actually be beneficial in providing a limit to the seismic base shear. Others design the structure for the overturning forces but ignore the tendency of the foundation to uplift. If the foundations are permitted to uplift, the engineer must investigate the redistribution of forces in the wall and in the soil due to the shift in the resultant of the soil pressure and also the potential distortion of structural and nonstructural elements framing into the wall.

3.6.2 INDIVIDUAL PIER OR COLUMN FOOTINGS

3.6.2.1 Deficiencies

The principal deficiencies in the seismic capacity of existing individual pier or column footings are:

- Excessive soil bearing pressure due to overturning forces,

- Excessive uplift conditions due to overturning forces, and

- Inadequate passive soil pressure to resist lateral loads.

3.6.2.2 Strengthening Techniques for Excessive Soil Bearing Pressure

Techniques. The problem of excessive soil bearing pressure due to overturning forces can be mitigated by:

1. Increasing the bearing capacity of the footing by underpinning the footing ends and/or providing additional footing area (Figure 3.6.1.2a).

2. Increasing the vertical capacity of the footing by adding new drilled piers adjacent and connected to the existing footing (Figure 3.6.1.2b).

3. Reducing the bearing pressure on the existing footings by connecting adjacent footings with deep reinforced concrete tie beams.

4. Increasing the soil bearing capacity by modifying the existing soil properties.

5. Reducing the overturning forces by providing supplemental vertical-resisting elements (i.e., shear walls or braced frames).

Relative Merits. The considerations in selecting alternatives to correcting excessive soil bearing pressure due to overturning forces in individual pier or column footings are similar to those discussed above for continuous or strip footings. There is, however, the additional alternative of tying adjacent footings together with a deep reinforced concrete beam (Technique 3), which may be a feasible means of distributing the forces resulting from the overturning moment to adjacent footings.

3.6.2.3 Strengthening Techniques for Excessive Uplift Conditions

Techniques. Deficient capacity of existing foundations to resist the prescribed uplift forces caused by seismic overturning moments can be improved by:

1. Increasing the uplift capacity of the existing footing by adding drilled piers or soil anchors.

2. Increasing the size of the existing footing to mobilize additional foundation and soil weight.

3. Increasing the uplift capacity by providing a new deep reinforced concrete beam to mobilize the dead load on an adjacent footing.

4. Reducing the uplift forces by providing supplemental vertical-resisting elements (i.e., shear walls or braced frames).

Relative Merits. The considerations in selecting techniques to correct excessive uplift conditions due to overturning forces in individual pier or column footings are similar to those discussed above for continuous or strip footings. Technique 2 is appropriate only when excessive uplift results from combined vertical loads and moments on the footing. There is, however, the additional alternative of tying adjacent footings together with a deep reinforced concrete beam (Technique 3), which may be a feasible means for mobilizing the existing mass supported by an adjacent footing.

3.6.2.4 Strengthening Techniques for Inadequate Passive Soil Pressure

Techniques. The problem of excessive passive soil pressure caused by seismic loads can be mitigated by:

1. Providing an increase in bearing area by enlarging the footing.
2. Providing an increase in bearing area by adding new tie beams between existing footings.
3. Improving the existing soil conditions adjacent to the footing to increase the allowable passive pressure.
4. Reducing the bearing pressure at overstressed locations by providing supplemental vertical-resisting elements such as shear walls or braced frames as discussed in Sec. 3.4.

Relative Merits. As noted above, foundation rework generally is relatively costly. The foundation strengthening technique that is the most cost-effective generally is the technique that can resolve more than one concern. The addition of a new deep tie beam between adjacent footings if required to resist overturning forces will likely address inadequate passive soil pressure concerns. As the above discussion indicates, the most cost-effective alternative to the strengthening of an existing foundation usually is not readily apparent. Several alternative schemes may have to be developed to the point where reasonable cost estimates can be made to evaluate the tangible costs (i.e., the total actual work that needs to be accomplished) as well as the disruption or relocation of an ongoing function and the architectural considerations.

3.6.3 PILES OR DRILLED PIERS

3.6.3.1 Deficiencies

The principal deficiencies in the seismic capacity of piles or drilled piers are:

- Excessive tensile or compressive loads on the piles or piers due to the seismic forces combined with the gravity loads and
- Inadequate lateral force capacity to transfer the seismic shears from the pile caps and the piles to the soil.

3.6.3.2 Strengthening Techniques for Excessive Tensile or Compressive Loads

Techniques. Deficient tensile or compressive capacity of piles or piers can be improved by:

1. Increasing the capacity of the foundation by driving additional piles and replacing or enlarging the existing pile cap (Figure 3.6.3.2).
2. Reducing the loads on overstressed pile caps by adding tie beams to adjacent pile caps and distributing the loads.

Relative Merits. Although it may be possible to drive additional piles to correct the deficiency, it usually is very difficult to utilize the existing pile cap to distribute the loads effectively to both old and new piles. It then may be necessary to consider temporary shoring of the column or other structural members supported by the pile caps so that the pile caps can be removed and replaced with a new pile cap that will include the new piles.

As discussed above for individual footings, it may be more cost-effective to provide deep tie beams to distribute some of the pile load to adjacent pile caps that may have excess capacity than to drive new piles.

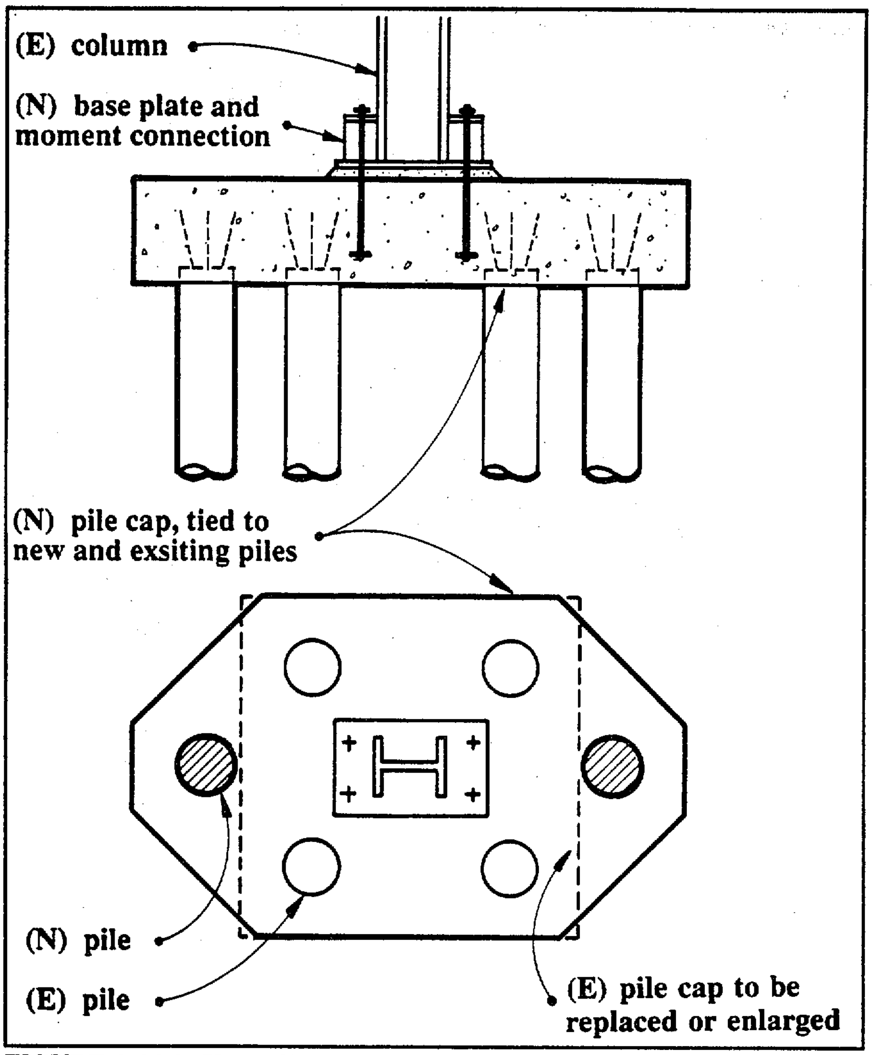

FIGURE 3.6.3.2 Upgrading an existing pile foundation.

3.6.3.3 Strengthening Techniques for Excessive Lateral Forces

Techniques. Deficient lateral force capacity of piles or piers can be improved by:

1. Reducing the loads on overstressed pile caps by adding tie beams to adjacent pile caps and distributing the loads.

2. Increasing the allowable passive pressure of the soil by improving the soil adjacent to the pile cap.

3. Increasing the capacity of the foundation by driving additional piles and replacing or enlarging existing pile cap.

4. Reducing loads on the piles or piers by providing supplemental vertical-resisting elements (i.e., braced frames or shear walls) and transferring forces to other foundation members with reserve capacity as discussed in Sec. 3.4.

Relative Merits. The most cost-effective approach may be to provide tie beams between piers or pile caps (Technique 1). The tie beams will distribute loads between foundation elements as well as provide additional surface area to mobilize additional passive pressure. In specific situations, the other alternatives may be more cost-effective depending upon accessibility as well as the impact each alternative may have on the ongoing functional use of the building.

3.6.4 MAT FOUNDATIONS

3.6.4.1 Deficiencies

Seismic deficiencies in mat foundations are not common; however, the following two deficiencies can occur:

- Inadequate moment capacity to resist combined gravity plus seismic overturning forces and
- Inadequate passive soil pressure to resist sliding.

3.6.4.2 Strengthening Technique for Inadequate Moment Capacity

Deficient mat foundation moment capacity can be corrected by increasing the mat capacity locally by providing additional reinforced concrete (i.e., an inverted column capital) doweled and bonded to the existing mat to act as a monolithic section. If the inadequacy is due to concentrated seismic overturning loads, it may be possible to provide new shear walls above the mat to distribute the overturning loads and also to locally increase the section modulus of the mat.

3.6.4.3 Strengthening Technique for Inadequate Lateral Resistance

Deficient mat foundation lateral resistance (e.g., the possibility of a mat founded at shallow depth in the soil) can be corrected by the construction of properly spaced shear keys at the mat perimeter. The shear keys would be constructed by trenching under the mat, installing dowels on the underside of the mat, and placing reinforced concrete in the trench.

3.7 DIAPHRAGM TO VERTICAL ELEMENT CONNECTIONS

Seismic inertial forces originate in all elements of buildings and are delivered through structural connections to horizontal diaphragms. The diaphragms distribute these forces to vertical elements that transfer the forces to the foundation.

An adequate connection between the diaphragm and the vertical elements is essential to the satisfactory performance of any structure. The connections must be capable of transferring the in-plane shear stress from the diaphragms to the vertical elements and of providing support for out-of-plane forces on the vertical elements.

The following types of diaphragms are discussed below: timber, concrete, precast concrete, steel deck without concrete fill, steel deck with concrete fill, and horizontal steel bracing.

3.7.1 CONNECTIONS IN TIMBER DIAPHRAGMS

3.7.1.1 Deficiencies

The principal connection deficiencies in timber diaphragms are:

- Inadequate capacity to transfer in-plane shear at the connection of the diaphragm to interior shear walls or vertical bracing,

- Inadequate capacity to transfer in-plane shear at the connection of the diaphragm to exterior shear walls or vertical bracing, and
- Inadequate out-of-plane anchorage at the connection of the diaphragm to exterior concrete or masonry walls.

3.7.1.2 Strengthening Techniques for Interior Shear Wall Connections

Deficient shear transfer capacity of a diaphragm at the connection to an interior shear wall or braced frame can be improved by:

1. Increasing the shear transfer capacity of the diaphragm local to the connection by providing additional nailing to existing or new blocking (Figure 3.7.1.2a).
2. Reducing the local shear transfer stresses by distributing the forces from the diaphragm by providing a collector member to transfer the diaphragm forces to the shear wall (Figure 3.7.1.2b).
3. Reducing the shear transfer stress in the existing connection by providing supplemental vertical-resisting elements as discussed in Sec. 3.4.

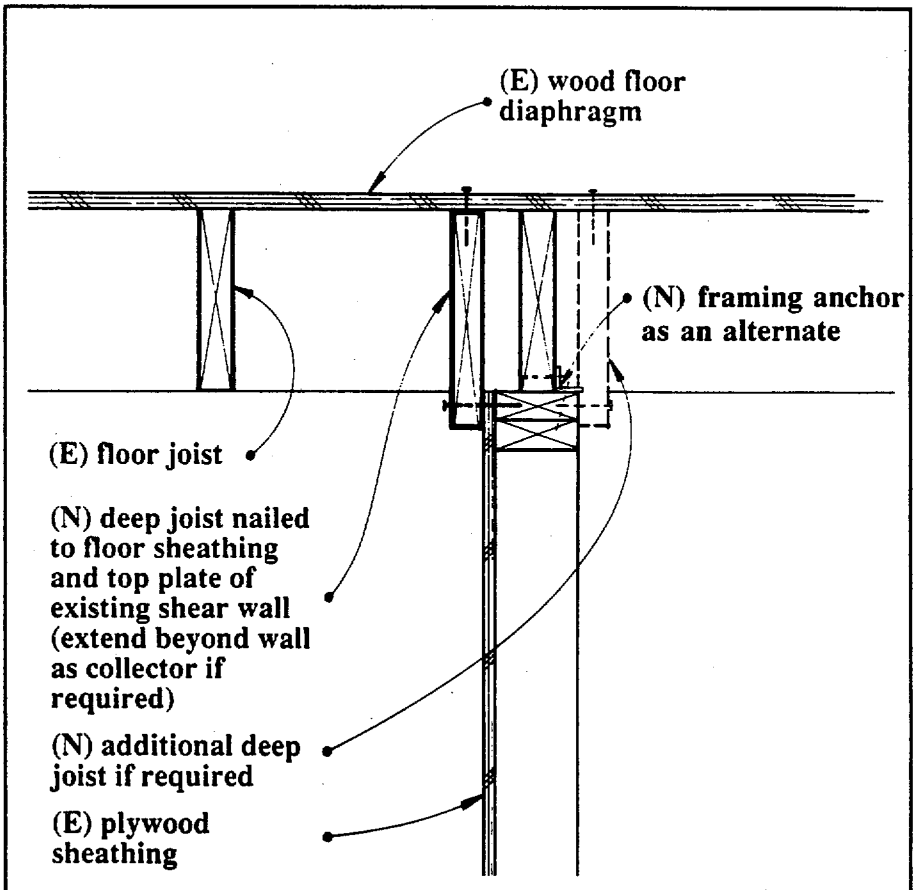

FIGURE 3.7.1.2a Strengthening the connection of a diaphragm to an interior shear wall (wall parallel to floor joist).